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Cyclic Undrained Behaviour of Silty Sand under Partial Cyclic Reversal Loading

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ABSTRACT: Cyclic undrained behaviour of sand with different fines content was investigated through stress-controlled cyclic triaxial tests with partial stress reversal. The test results were analysed based on the concept of equivalent granular state parameter, ψ*. Three different types of behaviour were observed: namely cyclic instability, cyclic mobility and an in-between transition behaviour. Cyclic instability was observed for loose sand-fines mixtures with a positive ψ*, and can be related to the instability stress ratio of monotonic test with same equivalent granular void ratio, *e** as long as instability was controlled by the compression side of the stress space. On the other hand, cyclic mobility was observed for samples had negative ψ*. However, transition behaviour was observed for samples with ψ* values close to zero.

1. INTRODUCTION

Liquefaction is a catastrophic failure phenomenon that leads to flow-like deformation. It has been investigated by many researchers in the last few decades. However, the main emphasis of those investigations were in clean sand although sand with fines (<0.075mm) is not uncommon in natural deposits. Recent studies show that the presence of fines changes soil behaviour considerably and should be treated accordingly when dealing with this type of soil [1]. Recently, there has been a surge of research interest in studying the liquefaction behaviour of sand-fines mixture soil using the critical state soil mechanics, CSSM, framework [1, 2, 3, 4, 5, 6, 7].

A literature review revealed that the liquefaction triggered by cyclic loading can be driven by mainly two different mechanisms namely cyclic mobility and cyclic instability. Ishihara [8] defined initial liquefaction as the occurrence of zero effective confining stress, and this occurred at about 5% double amplitude (DA) cyclic axial strain. However, the state of zero effective confining stress may be transient during a load cycle, and the cyclic stress can still be sustained for a few cycles after initial liquefaction. This type of cyclic liquefaction is referred later to as cyclic mobility. On the other hand, cyclic instability corresponds to the state where run-way deformation happening in conjunction with rapid pore water pressure generation and the effective stress path plummeting downwards. The prescribed cyclic stress cannot be sustained and a form of instability in the context of continuum mechanics occurred. This mechanism was observed in loose saturated soil which has a direct correspondence with flow liquefaction observed in monotonic loading [9]. Thus, many studies gave emphasis on the triggering of flow liquefaction i.e. static instability [1, 7, 10, 11] and their correspondence with cyclic instability [4, 12, 13].

Different approaches and parameters were used in the past to predict the triggering of instability. Sladen et al. [14] introduced the concept of collapse line defined as a straight line joining the peak point and Steady State (SS) point in a normalised effective stress space. This inherently assumed that ESP of specimens of the same void ratio and sheared under monotonic loading can be approximated by a single curve. Lade [15] introduced instability line, IL, as a line passing through peak points of undrained monotonic ESP though the origin of stress space. The concept of IL was also used by many to link monotonic and cyclic instability although it is important to note that IL is dependent on void ratio at start of undrained shearing. Hyodo et al. [12] illustrated that cyclic instability was triggered when cyclic ESP reached the instability region of corresponding monotonic test. Yamamuro and Covert [13] demonstrated that instability line derived from the undrained monotonic test also defined the triggering of cyclic liquefaction. Lo et al. [4] clearly demonstrated that instability stress ratio, η_{IS}, of monotonic test can predict the triggering of cyclic instability for one-way cyclic loading for sand with fines. The η_{IS} values determined from monotonic tests that changes with void ratio, *e* and thus, Chu and Leong [1] proposed a η_{IS}-*e* relation.

However, recent publication shows that void ratio is not an effective parameter in predicting behaviour of sand with fines. For a fines content less than a threshold value, f_{thre} , the sand-silt mixture still has a "fines-in-sand" matrix. Under this condition, Thevanayagam et al. [16] introduced equivalent granular void ratio, *e**, defines as:

$$e^{*} = \{e + (1-b) f_c\} / \{1-(1-b) f_c\} \tag{1}$$

where, f_c = fines content and *b* represents the fraction of fines that actively take part in the force structure of the sand-silt mixture, and therefore $1 \geq b \geq 0$. *e** was considered as an alternative to *e* because it can capture the effects of f_c. If we approximate b = 0 when f_c is small relative to f_{thre}, then Eq. (1) degenerates to the earlier concept of e_g as proposed by Thevanayagam [17]. But, the *b* values reported by many researchers were back-analysed values in order to achieve a intended single correlation [7, 18, 19]. This means *e** cannot be used as an input to a prediction unless the results are already known. To overcome this problem, Rahman et al. [6, 20]; Rahman and Lo [21] proposed the following equation for predicting *b* for $f_c<f_{thre}$.

$$b = \left[1 - \exp\left(0.3\frac{(f_c / f_{thre})}{k}\right)\right] \times \left(r\frac{f_c}{f_{thre}}\right)^{r} \tag{2}$$

where, $r = \chi^{-1} = (D_{10}/d_{50})^{-1} = d_{50}/D_{10}$, $k = (1 - r^{0.25})$. D_{10} = size of sand at 10% fractile and d_{50} = size of fines at 50% fractile. To determine f_{thre}, Rahman et al. [20]; Rahman and Lo [21] also proposed an equation as:

$$f_{thre} = A\left(\frac{1}{1+e^{\alpha-\beta\chi}} + \frac{1}{\chi}\right) \tag{3}$$

The coefficient of *A* is the asymptotic value of 0.40. Based on a comprehensive review of published data, Rahman and Lo [21] inferred that α =0.50 and β =0.13. Using above proposed model, the SS data point as plotted in a *e**-log(p') space can be described by a single trend, which is termed as equivalent granular steady state line, EG-SSL [6, 20].

The study aims to use *e** and a new concept proposed in a later section to predict three types of liquefaction behaviour namely cyclic instability, cyclic mobility and in-between transition behaviour. The correspondence between cyclic and monotonic instability was also investigated.

2. HYPOTHESIS

The state parameter, ψ, originally proposed by Been and Jefferies [22] is considered as a good parameter to predict clean sand behaviour. But, to capture the effect of fines, Rahman and Lo [23] modified the original definition by replacing e with e^* and SSL with EG-SSL, thus defining an equivalent granular state parameter, ψ^* as in Eq. (4).

$$\psi^* = e^* - e^*_{SS} \tag{4}$$

It was shown that η_{IS} can be correlated to $\psi^*(0)$, ψ^* at start of shearing, irrespective of fines content as illustrated in Fig. 1. We hypothesized that $\psi^*(0)$, might be a predictor of cyclic liquefaction mechanisms.

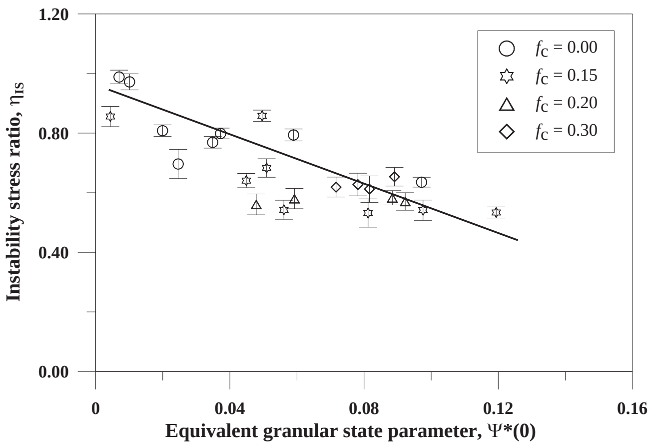

Figure 1 Relationship between η_{IS} and $\psi^*(0)$ for different % of f_c; after Rahman and Lo [23]

3. EXPERIMENTAL STUDY

3.1 Tested Material

Uniform size quartz sand (SP) called Sydney sand and well-graded low plasticity fines (PI=27, LL=54) were used in this study. Fines content in this study was in the range of 15-30% by dry weight. Physical properties and grading curve of tested material can be found in [24].

3.2 Experimental Procedure

A series of stress controlled undrained cyclic tests were performed. The cyclic loading can have any combination of peak and trough deviator stress, and commencing from any static stress state. An internal load cell was used to illuminate any friction induced from external loading ram. One pair of internal Linear Variable Differential Transformers (LVDT) attached to the top platen was used to calculate axial strain at initial stage of shearing. Another external LVDT was used at later stage when these two went out of limit. Cell and pore pressure was controlled by large and small Digital Pressure Volume Controller (DPVC) respectively.

The specimens were 100 mm diameter x 100 mm height and tested with free ends and enlarged platen [25] to reduce end restraint. It was prepared by moist temping method compacting 10 uniform layers with predetermined amount of moist soil. Liquid rubber was also used to minimise bedding and membrane penetration error.

4. TEST RESULTS AND DISCUSSION

The test conditions covered: initial effective confining stress, p_0', 200 to 850 kPa; void ratio, e, 0.429 to 0.678; f_c, 15 to 30%, and e^*, 0.638 to 0.922. These initial conditions were plotted in e^*-log(p'_0) space as shown in Fig. 2. Tests manifesting cyclic instability are plotted with "open" symbols. These points are all located above the EG-SSL which corresponds to $\psi^*(0) > 0$. On the other hand, test manifesting cyclic mobility are plotted with "partial filled" symbols. These points are all located below the EG-SSL which corresponds to $\psi^*(0) < 0$. There are four data points shown by "fully filled" symbols which are located very close to the EG-SSL, a $\psi^*(0)$ in the range of –0.012 to +0.015. These data points correspond to tests showing a transition form of liquefaction. Therefore, three form of cyclic liquefaction can be predicted by the values of $\psi^*(0)$.

Detailed behaviour of these three mechanisms of cyclic liquefaction will be presented in the following sub-sections. Due to page limitation, only one representative test for each mechanism will be presented.

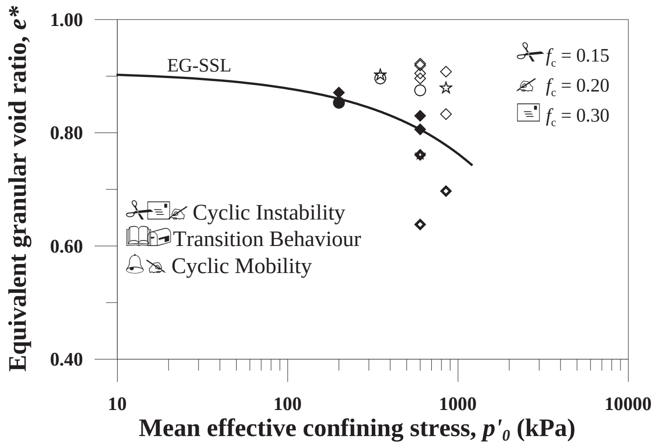

Figure 2 Initial conditions of all tests for sand with different % of f_c

4.1 Cyclic Instability Behaviour for $\psi^*(0)>0$

Figure 3a shows the cyclic ESP whereas Fig. 3b represents q-ε_1

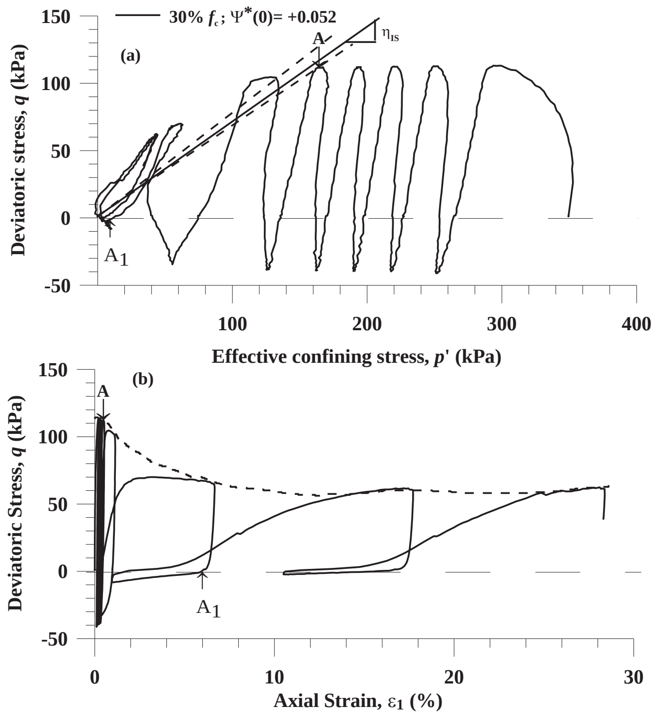

Figure 3 Relationship between monotonic and cyclic instability for $\psi^*(0) = +0.052$ with 30% f_c; (a) ESP; (b) q-ε_1 plot

response. The initial condition of this test was: $\psi^*(0) = +0.052$ conducted at $p_0' = 350$ kPa. The peak, q_{peak} and trough, q_{min} q-values (of 112 kPa and -39 kPa respectively) were chosen so that potential of cyclic instability was manifested in the compression side of p'-q space. In the first 5 load cycles, ESP moved left side with loading cycles and the leftward movement per cycle was getting smaller. From the 6th cycle onwards, leftward movement of the ESP started to increase with load cycles and developing large axial strain (Fig. 3b). The prescribed q_{peak} value cannot be developed rather plummet downwards. Furthermore, the q_{peak} value attained was reducing with load cycles. Thus, cyclic instability occurred because if a maintained q value was imposed, then run-away deformation would occur. After point A, q_{peak} of ESP drop quickly and then remained same at the end of the test indicates SS (Fig. 3b). Thus, point A may be approximated as the triggering point of cyclic instability. The two dotted lines shown in Fig. 3a correspond to the estimated range for defining onset of strain softening. This range can be compared with η_{IS} as determined from monotonic loading inferred from the η_{IS}-$\psi^*(0)$ of Fig. 1. This value is denoted by the slope of the solid straight line through the origin shown in Fig. 3a. Evidently, the two dotted lines are closed to both sides of the solid line. It should be noted that there is also some possible variation in determining η_{IS} from Fig. 1. Therefore, simply by knowing $\psi^*(0)$, one can predict the onset of cyclic instability.

The above finding is applicable to tests conducted with different combinations of q_{peak} and q_{min} as long as $\psi^*(0)>0$ and cyclic instability occurred in the compression side of stress space.

4.2 Cyclic Mobility Behaviour for $\psi^*(0) < 0$

Cyclic mobility behaviour is discussed in this section through the experimental evidence. The test was conducted at p_0' of 600 kPa with $\psi^*(0)$ = -0.040 for sand with 20% fines content. Figure 4a shows the ESP whereas Fig. 4b shows q-ε_1 response. ESP had a q_{peak} and q_{min} value of 215 kPa and -138 kPa respectively. At the start of shearing, the leftward movement of cyclic ESP was faster up to 8

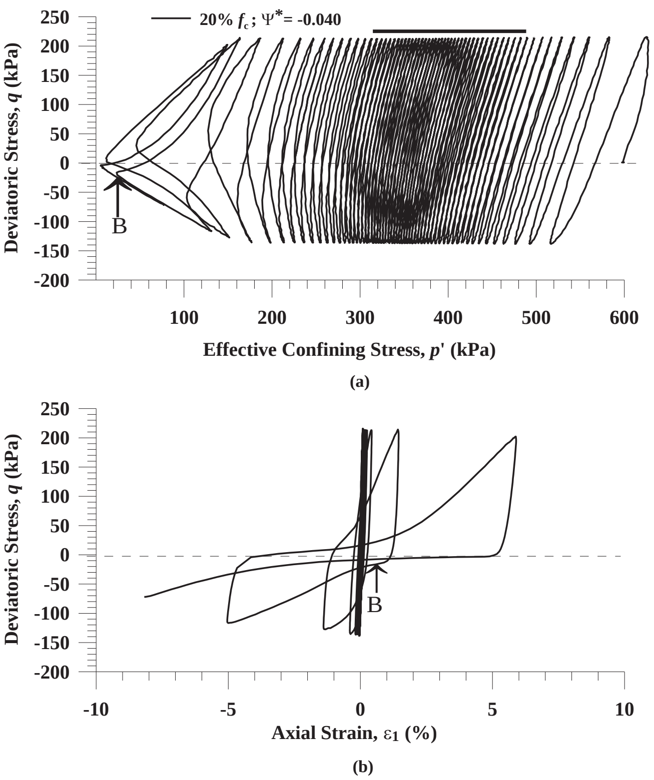

Figure 4 Cyclic mobility behaviour for $\psi^*(0)$ = -0.040 with 20% f_c; (a) ESP; (b) q-ε_1 plot

cycles with slight inclination to the right. But in next 37 cycles (marked by thick solid straight line on top in Fig. 4a), ESP was moved very slowly left side and started to bend. After passing this region, ESP again started to move left side faster where complete crossing occurred during load cycling. Point B in Fig. 4a-b indicates 95% pwp generation and very close to zero effective stress. Thus, this point can be considered as occurrence of cyclic mobility. After this point, the deviator stress was maintained held at a constant value slightly less than q_{peak}. Also, no run-away deformation was observed while travelling through near zero effective stress. Therefore, this is not a form of instability. Similar observation was reported for all other tests with $\psi^*(0)$ value as negative.

4.3 Transition Behaviour for $\psi^*(0) \approx 0$

In between transition behaviour other than cyclic instability and cyclic mobility is discussed in this section. The soil sample was tested at p_0' of 600 kPa with $\psi^*(0)$ = +0.015 for 15% f_c. Figure 5a represents ESP whereas q-ε_1 response shown in Fig. 5b. The q_{peak} and q_{min} was 209 kPa and -66 kPa respectively. At the start of shearing, ESP moved left side without that much inclination as observed previously in Fig. 4a. But, ESP started to bend and cross slightly in the middle as load cycles progressed (Fig. 5a). However, ESP did not cross completely each other at the end of the test. The point C in Fig. 5a-b corresponds to zero effective stress with 99% pwp generation. Therefore, this point can be considered as the occurrence of liquefaction. Thereafter, soil sample maintained almost same q_{peak} but q_{min} reduced dramatically close to zero value. No strain softening was noticed though strain accumulation was high after point C. This behaviour is somewhat different than cyclic mobility as observed in Fig. 4. Therefore, it is termed as transition behaviour.

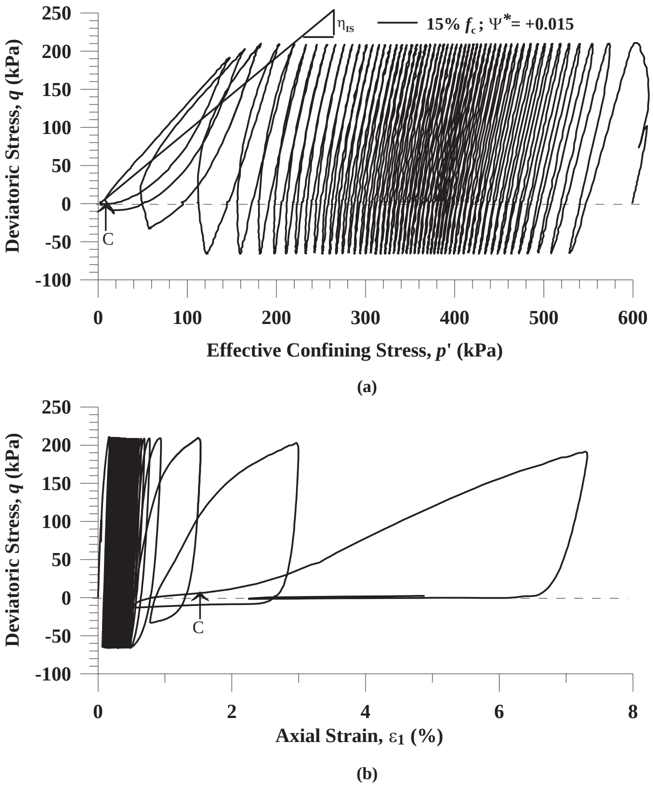

Figure 5 Transition behaviour for 15% f_c with $\psi^*(0)$ = +0.015; (a) ESP; (b) q-ε_1 plot

5. CONCLUSION

A series of undrained triaxial tests were performed on sand with 15-30% fines to investigate cyclic undrained behaviour under partial cyclic reversal loading. A range of p_0' is also covered. The significant findings observed from the study are summarized as follows:

(a) Cyclic liquefaction mechanisms can be characterized in three groups; cyclic instability, cyclic mobility and transition behaviour. These three forms of cyclic liquefaction can be predicted from equivalent granular state parameter at start of cyclic shearing.

(b) Triggering of cyclic instability can be predicted from the η_{IS} - $\psi^*(0)$ relationship pre-established from monotonic shearing to instability.

(c) After the occurrence of cyclic instability and transition behaviour in the compression side, the deviator resistance in the extension was reduced to a very small value.

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